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An Investigation Of The Causes And Recommendations For Correction Of The Pinole Slipout, Road 04-CC-80

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16. ABSTRACT

This report covers an engineering study of an earth fill failure which occurred May 11, 1969, resulting in the closure of Interstate Route 80 near Pinole. It is offered in compliance with House Resolution No.229. Immediately before the failure, traffic was rerouted to adjacent city streets and county highways. Construction of a detour we then started on State right of way adjacent to the slipout.

An engineering investigation was also started to determine the cause of the failure and to provide information on which to base corrective action.

It was found that the failure was caused by the accumulation of an excessive volume of groundwater in the original ground under the highway fill. This groundwater was under pressure, which weakened the foundation soils and resulted in failure.

The fill was constructed in 1957 under a contract which was awarded in January, 1956 and completed in April, 1958. The subsurface character of the ground over which the fill was built was explored in 1954. At that time, no significant groundwater was discovered. The foundation was judged to be competent for the fill to be constructed, and it is the present findings that if it were not for the excessive accumulation of water as the result of this past winter's storms, the foundation would still be appropriate for this fill. To some small extent the construction of earth fills for a subdivision along the downstream side of the highway appear to have contributed to the accumulation of water under the fill. This was due primarily to a restriction to both surface and subsurface drainage.

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STATE OF CALIFORNIA

DEPARTMENT OF PUBLIC WORKS

AN INVESTIGATION OF THE CAUSES
and
RECOMMENDATIONS FOR CORRECTION
of the
PINOLE SLIPOUT
ROAD 04-CC-80

HOUSE RESOLUTION NO. 229

1969 LEGISLATIVE SESSION

JUNE 1969

69-29

DND

STATE OF CALIFORNIA—TRANSPORTATION AGENCY
RONALD REAGAN, Governor

DEPARTMENT OF PUBLIC WORKS
1120 N STREET
SACRAMENTO, CALIFORNIA 95814

June 13, 1969

Hon. James D. Driscoll
Chief Clerk of the Assembly
State Capitol
Sacramento, California

Dear Mr. Driscoll:

Pursuant to the provisions of House Resolution No. 229 (Knox - 1969), I am transmitting a report entitled "An Investigation of the Causes and Recommendations for Correction of the Pinole Slipout, Road 04-CC-80."

Sincerely,

James A. Moe

JAMES A. MOE
Director of Public Works

Attachment

DEPARTMENT OF PUBLIC WORKS
1120 N STREET
SACRAMENTO, CALIFORNIA 95814

June 13, 1969

Hon. C. D. Alexander
Secretary of the Senate
State Capitol
Sacramento, California

Dear Mr. Alexander:

Pursuant to the provisions of House Resolution No. 229 (Knox - 1969), I am transmitting a report entitled "An Investigation of the Causes and Recommendations for Correction of the Pinole Slipout, Road 04-CC-80."

Sincerely,

James A. Moe

JAMES A. MOE
Director of Public Works

Attachment

1969 Legislative Session

HOUSE RESOLUTION NO. 229

The causes of a fill slide on Interstate Route 80 near Pinole.

" * * * *

"Resolved by the Assembly of the State of California, That the Members request the Director of Public Works to submit a preliminary report to the Legislature within 30 days as to the cause of the slide on Interstate Route 80 near Pinole, and to indicate therein when a final report will be submitted;..."

Memorandum

To : Mr. James A. Moe
Director of Public Works

Date : June 16, 1969

File : House Resolution No. 229
Reference: 32082

From : **Department of Public Works--Division of Highways**

Subject:

Submitted herewith are copies of the report entitled "Investigation of the Causes and Recommendations for Correction of the Pinole Slipout, Road 04-CC-80." This report was prepared pursuant to House Resolution No. 229 (copy included in each report) introduced during the 1969 Legislative Session.

Briefly summarized, the embankment slipout was the result of a strength loss of the foundation soils due to an unprecedented rise in water table elevation. This rise was due to an extremely severe series of winter rain storms, both in terms of total precipitation and number of rain days. In addition, the construction of subdivision fills downslope of the slide area eight years after construction of the embankment apparently contributed to the accumulation of water under the fill by restriction of both surface and subsurface drainage.

The correction will consist of the construction of an extensive subdrainage system in order to effect a permanent lowering of the water table within the embankment foundation. For increased embankment stability, a berm will be constructed laterally a distance of at least 50 feet beyond the limit of downhill movement.

This investigation indicates the cause of the slide to be primarily the accumulation of storm water under the fill. We have therefore applied for a 100% financing of the reconstruction under the 1969 Emergency Relief Program. Our application is presently being processed by the Bureau of Public Roads.

J. A. LEGARRA
State Highway Engineer

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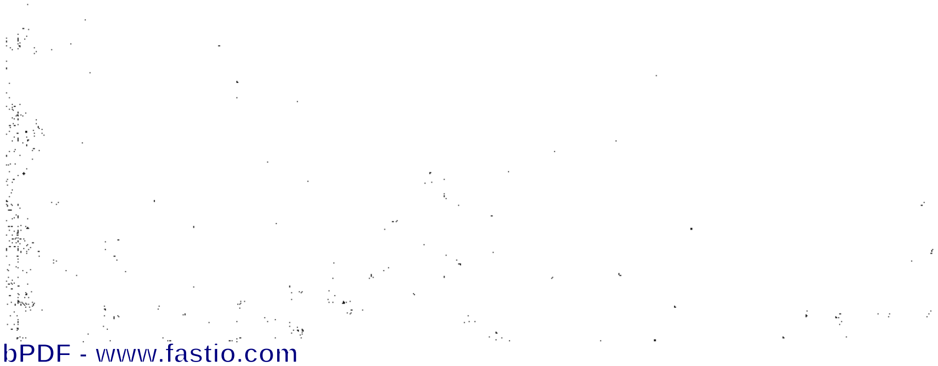

SYNOPSIS

This report covers an engineering study of an earth fill failure which occurred May 11, 1969, resulting in the closure of Interstate Route 80 near Pinole. It is offered in compliance with House Resolution No. 229. Immediately before the failure, traffic was rerouted to adjacent city streets and county highways. Construction of a detour was then started on State right of way adjacent to the slipout.

An engineering investigation was also started to determine the cause of the failure and to provide information on which to base corrective action.

It was found that the failure was caused by the accumulation of an excessive volume of groundwater in the original ground under the highway fill. This groundwater was under pressure, which weakened the foundation soils and resulted in failure.

The fill was constructed in 1957 under a contract which was awarded in January, 1956 and completed in April, 1958. The subsurface character of the ground over which the fill was built was explored in 1954. At that time, no significant groundwater was discovered. The foundation was judged to be competent for the fill to be constructed, and it is the present findings that if it were not for the excessive accumulation of water as the result of this past winter's storms, the foundation would still be appropriate for this fill. To some small extent the construction of earth fills for a subdivision along the downstream side of the highway appear to have contributed to the accumulation of water under the fill. This was due primarily to a restriction to both surface and subsurface drainage.

At present, water is being removed from the fill by a series of drilled-in horizontal drains so as to stabilize the detour fill. Replacement construction will consist of a comprehensive subsurface drainage system followed by rebuilding the fill in its same position and reinforcing it with a counterweight fill along its downstream side.

Upon completion of this work, which will be under contract, a final report will be submitted.

I. INTRODUCTION

During the 1969 session of the Legislature, the Assembly passed House Resolution No. 229. This resolution requested that the Director of Public Works submit a preliminary report to the Legislature within 30 days as to the cause of the slide on Interstate Route 80 near Pinole. Personnel of the California Division of Highways have prepared the following report on the cause of the above-referenced highway slipout. Subsequent sections of this report will cover original exploration, detour, construction, geology, recent exploration, laboratory tests, analysis of test data, conclusions as to the cause of the slipout and a plan for correction. A final report with details on the design and construction of the correction will be submitted upon termination of the contract.

The subject embankment is located between Hilltop Drive and Appian Way and more particularly, between Stations 204± and 210± ,(PM 7.0±). It crosses a natural northerly bearing drainage swale in a northeasterly direction. Maximum embankment height is 60 feet at roadway centerline, 84 feet at the north toe and 46 feet at the south toe. The original embankment top width was 106 feet wide with 2:1 sideslopes. The roadway is a 6-lane portland cement concrete paved divided facility with a 12 foot median and 8 foot shoulders constructed under Contract 57-4TC1-F between January 1956 and April 1958.

Slide Sequence

Minor cracking and settlement first appeared on Tuesday, May 6, 1969, primarily in the outside westbound lane (toward San Francisco) at the location of the present slide. Work was done to level the pavement by means of mudjacking (pressure grouting underneath the concrete pavement slabs) and, as settlement continued, courses of asphalt surfacing were placed to level the pavement and keep it open to traffic.

This procedure was continued during the week until Sunday morning, May 11, when the slide began to accelerate and the serious nature of the failure became apparent. At 11:00 AM it was decided that the road should be closed by noon. Within that hour the Highway Patrol was notified, radio stations informed, the Carquinez Bridge was requested to alert southbound vehicles, and cities and police forces were alerted to prepare for unusual traffic on alternate routes. In addition, Division Maintenance forces manufactured and erected directional signing and provided illumination for these signs. At noon it was necessary to close all westbound lanes and two of the three eastbound lanes. The only lane remaining open was the outside eastbound lane.

The slide continued northerly during Sunday and between 5:00 PM and 6:00 PM ruptured 6" and 8" product lines owned by Union Oil Company and Southern Pacific Pipe Lines Company, respectively. These lines were located outside and parallel to the freeway right of way in an easement and the 6" line was carrying, at the time, aviation fuel. The fuel flowed from the ruptured lines downhill into the storm drain system of an adjacent [illegible] Associate [illegible]

[illegible] following recent [illegible] of test data, concentrations as [illegible] final report with [illegible] will be submitted.

[illegible] is located between [illegible] Drive [illegible]

[illegible]

[illegible]

[illegible]

subdivision. The Pinole Fire Department responded immediately and applied chemical foam as a precaution against possible fire. Minor sloughing on the west escarpment face of the slide continued after the major movement took place on Sunday, May 11, to a point where, in the interest of public safety, it was best not to route traffic adjacent to the slide area. The single northbound lane was closed at 9:45 AM on Wednesday, May 14, thereby completely closing all six lanes in both directions to traffic. An overall view of the slide is presented by Figure 1.

Detour Construction

Detour work started Sunday afternoon, May 11, 1969. That day contact was made with a major well qualified construction firm, Gordon Ball Enterprises, which was currently doing work on nearby Route 24 in the Orinda area. Accordingly, they began delivering heavy road building equipment to the site from the Orinda area and from other going projects as far away as Byron. A design plan and cross-sections were prepared by Monday morning, May 12, providing for a 6-lane bypass on the south side of the slide area that would be capable of handling freeway traffic until the exact cause of the failure was determined and permanent corrective construction could take place.

The detour provided was approximately 1/3 mile long, complete with six 12' wide lanes, a 12' wide median and shoulders. The primary objective after the slide occurred was to make an all out effort to restore traffic service at the earliest possible time. The Director of Public Works approved an emergency force account contract with Gordon Ball Enterprises on May 11. It provided for work and materials to construct the estimated $300,000 detour. This involved excavating and placing 100,000 cubic yards of material. Approximately 27 pieces of heavy road building equipment was used during this phase of the work. In all, some 400 men were involved on the job for both the Contractor and the Division of Highways working with alternate crews on a 24-hour basis. Seventy-five trucks were used to haul 12,000 tons of asphalt and 20,000 tons of aggregate base material.

By 11:00 PM Friday, May 16, one southbound lane of the detour was opened. On Saturday, May 17, at 10:30 PM one northbound lane was opened. On Sunday, May 18, two more southbound lanes were opened at 11:30 AM. At 1:30 PM the two remaining northbound lanes were opened, thus the freeway was open to full traffic service approximately one week after the first lane closing.

Project History

The construction of the portion of what is now Interstate Route 80 between the North City Limits of San Pablo and the Carquinez Bridge was completed in 1958. This route is the principal highway for the east bay north of El Cerrito. It is a portion of the main transcontinental system providing all year connection with areas east of the Sierra Nevada mountains and is also a highway of military importance.

Completion of this route relocation marked the culmination of planning that began in 1928. The route which it replaced had been an old county road which was included in the state highway system when routes were initially established. Beginning in 1914, numerous improvements in alignment, [illegible] 1969. Thursday [illegible]

[illegible] on nearby Route 24 at the [illegible]

[illegible] were prepared by Monday morning [illegible] the south side of the slide [illegible]

[illegible] construction [illegible]

[illegible] approximately 1/3 mile long [illegible] shoulders [illegible] The Director of [illegible] Division [illegible] to contract [illegible] 100,000 cubic yards of materials [illegible] equipment [illegible] working with [illegible] able to haul 12,000 [illegible] material.

[illegible]

[illegible] the portion of [illegible] and the [illegible] the principal highway [illegible] transcontinental [illegible] part of the Sierra Nevada [illegible]

[illegible] relocation marked the culmination of [illegible] the route which it replaced [illegible] in the state highway system [illegible] in 1916, numerous improvements [illegible] grade, width, and surfacing had been made. The last major improvement came during and shortly after World War II when four 10-foot lanes were provided as a "temporary" expedient.

In the late 1940's, planning efforts were intensified. The project report for this 8.9 mile long relocation was submitted in April, 1952. For administrative purposes, the project had been split into two portions. Each portion was covered by separate materials and preliminary reports and separate contracts were eventually let. The southwesterly portion in which the subject slipout occurred ran from south of Hilltop Drive to just beyond the North City Limits of Hercules, a distance of almost five miles. The materials report for this section was submitted in September, 1954.

Original Exploration and Findings

The original exploration for the southwesterly portion was made during the first half of 1954. Ninety borings were made with the 1" California type soil sampler.

This device provides information on soil classification, density, moisture content, and resistance to penetration. It does not take samples suitable for strength and settlement testing. It is usually employed, as it was in this case, to evaluate the suitability of fill or drainage structure foundation materials by a determination of the extent and depth of loose soil mantle or soft compressible materials. On this project, it was utilized for the investigation at nine proposed fill areas.

Twenty-four power borings were also made. These were auger borings and were mainly used to obtain test samples from nine proposed cut areas. Samples obtained in this manner, although disturbed, can be used to evaluate the suitability of the material for use in embankment. Power borings were also used on this project to evaluate fill foundations in areas where the foundation was suspect.

In addition, test holes were dug at grade points and the information obtained was used to assist in estimating shrinkage factors for construction quantities. Visible evidence of the soil profile or structure, such as might be provided by existing cut banks or exposures on or adjacent to the new line, were duly examined, noted, and considered in the exploration effort. Also noted and considered were areas which were wet, as evidenced by vegetal growth type or other phenomena, and areas where local topography indicated previous instability.

A separate soils investigation was made at all proposed bridge abutment sites. The investigation at each site varied in complexity depending on the nature of the soil conditions encountered. In general, these site investigations were comprised of power driven cone penetrometer borings supplemented by sample borings where necessary.

Finally, the proposed route was traversed on foot and a field review was made by an experienced soils engineer to ascertain the extent and adequacy of the exploration.

A comparison of the exploration effort for the various cut and fill areas on the project is shown in Figure 2. Here, power borings and 1" soil probe borings are differentiated.

The 1" soil probe borings taken in the fill areas revealed a general pattern of two to three feet of black plastic clay loam over brown silty and sandy clay loams. Water was seldom encountered. Those borings made in existing drainage courses generally indicated wet plastic clays of variable depth, which would require removal. In most locations, there was no need for special treatment.

In the vicinity of the recent slipout, two 1" borings were made on the sidehill to the left of Stations 203± and 204±. These borings were limited, by high resistance to penetration, to depths of 6' and 4', respectively. They indicated no special problems. At Station 207±, two 1" borings were made in a small rounded "valley. One boring, to the left of centerline, went 7' and the other, on the right of centerline, went to a depth of 5'. In both cases, the borings were stopped by high resistance to penetration. They showed a moisture content of 30% existing in the black, clay loams. It was recommended that a minimum of 3' of unsuitable material be removed from the culvert foundation area. Two borings at Station 210±, one on the right side and one on the left side of centerline went to a depth of 4' before resistance to penetration terminated the borings. No problems were evident in this area. The borings in this vicinity were made May 7 - 10, 1954.

Power borings, as shown by Figure 2, were generally made only in proposed cut areas. However, in two proposed fill areas, (Station 346 - 19+75 and 30+50 - 45±) where a lush growth of grass made moisture conditions suspect, power borings were made. These borings revealed the existence of compressible soils underlain in some areas by soils of lower strength. Because of this, these two fill areas were further explored with 12 two-inch sample borings at the start of construction. At this time, instrumentation was installed to monitor settlement and and pore water pressure during construction of the fills.

Non-Highway Related Construction

On April 6, 1964, the firm of Woodward-Clyde-Sherard and Associates, Consulting Soil and Foundation Engineers, submitted a report entitled "Soil Investigation for the Proposed Tara Hills Subdivision Unit #8." This tract directly abuts the west U.S. Highway 80 right of way line in the slide area. A review of the above-referenced report, indicates grass is the only form of vegetation with no evidence of past landslide activity in the immediate vicinity. A review of the boring data from three borings contiguous to the slide area revealed, the same general soil profile as that developed from the preliminary highway investigations of the fill and from the borings made subsequent to the slipout. The soils observed were of two to five feet of dark brown silty sandy clay underlain by stiff to very stiff, tan to brown silty and sandy clay followed by hard claystones and clayey sands at 2 to 9 feet. It was noted that the weaker and more compressible materials occurred at greater depths in the low lying drainage course over which the fill was constructed. Of the three borings in the immediate vicinity of the slide, water was found in only one. At their Boring No. 11 (Elevation 112) seepage was noted at 6 feet below the ground surface.

Earthwork fill for the subdivision was accomplished from May through July, 1964. The design involved the construction of two fills approximately 25 feet in height in a westerly and northwesterly direction across the drainage

channel which converged approximately 380 feet from the right of way line. The possibility of a raise in the water table as a result of subdivision fill construction is indicated by a series of aerial photographs taken at the site between 1956 and the present (Figure 3). In the 1956 photo, prior to the construction of either the subdivision or the highway, the terrain has a smooth and barren appearance. In 1961, four years after construction of the fill, but prior to the construction of the subdivision, we note very little change in the appearance of the terrain with the exception of fill slope vegetation. In 1965, one year after the construction of the subdivision fills, there is evidence of an increase in both highway embankment and subdivision fill slope vegetation. The 1969 photograph taken immediately after the occurrence of the slide in May, reveals areas of relatively lush vegetation at the base of the subdivision fill slopes. This more recent vegetation is shown more clearly in relation to the subdivision fill slopes by Figure 1.

II. SLIDE EXPLORATION

Geology

The slipout is located in rolling, grass covered hills of the Coast Range. Rock outcrops are sparse. The best exposures are the adjacent highway cuts.

About two to five feet of soil has been developed on the underlying material. This soil is black, organic and plastic. Analyses by X-ray diffraction and DTA indicate the presence of montmorillonite and organic material.

The material underlying the slipout consists of fine-to coarse-grained sandstones, siltstones, claystones and some conglomerates. For the most part, the sandstones are friable and the claystones are soft. These nonmarine sediments are probably part of the Orinda formation.

The structural trend is northwest-southeast. The beds exposed in the cut immediately south of the slipout strike northwest and dip 60° to 70° to the northeast. The beds exposed in the cut east of the slipout strike NW and dip about 35° to the SW. This change in direction of the dip can be explained either by folding or by faulting. In either case, groundwater would tend to be concentrated beneath the slipout.

The northwest trending creek makes an unusual loop in the vicinity of the fill. This loop is probably controlled by the geologic structure.

Precipitation Data

Precipitation data for the Pinole Slide area was recorded at United States Weather Bureau stations at Richmond and Crockett. An evaluation of the precipitation recorded during the existence of the freeway shows that the rainfall for the 1968-1969 winter season was unusually high. At Rich-mond,the amounts recorded during 2, 3,4,and 5 month periods were the high-est in the 11 years and during the one and 6 month periods, the second high-est. At Crockett, the amounts recorded during a 4 month period was the highest in 11 years. During 2, 3, 4, 5, and 6 month periods the second highest and during a one month period the third highest. As shown by Table 1, the bulk of the precipitation which occurred during this past season was recorded during the winter months from November through February. During the other two wet seasons (1962-63 and 1966-67), however, a comparatively larger percentage of precipitation occurred during the warmer months of October, March and April when evaporation rates are higher and infiltration rates lower. While no U.S. Weather Bureau or county precipitation gages are located in the immediate vicinity of the Pinole Slide, it is reasonable to assume that the data for the above-referenced two stations are indicative of the actual precipitation amounts to which the site was subjected. The Rich-mond data can be considered to be reasonably representative of the slide area.

Figure 4 presents a plot of number of rain days (1/10 inch or more per day) during the six month winter period of October through March vs. year since 1953. It is believed that number of rain days which occur during the winter season is probably a better indicator of infiltration of precipitation than total precipitation for a given period, since, very frequently, relatively [illegible] Richardson? [illegible]

[illegible] A [illegible] by X-ray [illegible] and [illegible]

[illegible] (Generally [illegible]

[illegible] the [illegible]

[illegible] the alternative [illegible] and [illegible] the [illegible] changes in [illegible]

[illegible] probably controlled by the geologic structure. [illegible]

[illegible] the Pinole Side area was used [illegible] in Richmond [illegible] the existence of [illegible] winter [illegible] months [illegible] the [illegible] during dry [illegible] periods [illegible] the [illegible] dry season [illegible] however [illegible]

[illegible] month of October [illegible] which [illegible] of precipitation [illegible] light but steady rainfall over an extended period of time will result in less surface runoff than would be the case for intermittent but extremely heavy or intense rains even though the total amount of precipitation might be the same. Here we note what may be considered average years, during the initial investigation (1954) and construction (1956). The winter season 1957-58, one year after construction, produced an unusually large number of rain days. From 1957-58 through 1967-68, we note a relatively dry period with respect to number of rain days. The winter period 1968-69 produced the highest number of rain days (53) for the period under consideration. It is interesting to note, however, that the winter period one year after construction of the embankment (1957-58) with 45 rain days closely approached the record 1968-69 winter season with no apparent visible distress on the embankment or pavement. The only significant difference between these two seasons, therefore, was the existence of non-highway related construction in the natural drainage swale produced by the fill subdivision that came into existence in 1964.

Field Exploration

Two drill rigs and crews were moved into the slipout area within hours after the earth movement occurred, arriving on the job on May 12, 1969.

A total of 12 borings were made in the area with a bucket auger unit by May 21, 1969 which are designated by the prefix letter of "P" on the attached plan (Figure 5).

The second drill rig and crew drilled 9 sample borings with inclinometer * installations in 7 of the borings. This work was completed on May 23, 1969. On May 18, a third drill crew and rig was moved to the area. This crew and equipment worked through May 29, 1969, completing 5 sample borings. Inclinometers were installed in two of these borings. The test borings made by the third unit is designated by the prefix letter "D" and the inclinometer installations by the prefix letter of "I".

Six additional sample borings were also made in the area by a fourth crew working through the weekend between May 16, 1969 and May 19, 1969. These borings are designated by the prefix letter of "B".

In summary, a total of 36 vertical test borings were made for a total of 1622 lineal feet of vertical hole. Inclinometer installations were made in 9 of the borings totaling 660 lineal feet. The logs of 7 borings considered representative of slide area are presented by Figures 6-13, incl.

By Monday, May 19, 1969, it was evident that a large amount of groundwater was present in the slipout area. The early borings revealed groundwater at from 20 to 25 feet from the ground surface in the area near the toe of the detour embankment which would rise from 10 to 15 feet upon being encountered. Groundwater near the toe of the slide was observed at original ground level and under a slight amount of head.

*A device with which it is possible to determine the depth of subsurface movement.

In order to provide immediate relief, 18" diameter slotted steel pipes were inserted in 2 of the 3 foot diameter bucket holes in the vicinity of the detour embankment toe. These were subsequently backfilled with permeable material and pumped dry six times a day. In addition, immediate steps were taken to have a contract let for the installation of horizontal drains. A private horizontal drilling contractor moved a crew and equipment into the lower part of the slipout and commenced horizontal drain installation work on Thursday, May 22, 1969. To date, ten horizontal drains have been installed. These were 700 to 800 feet in length and several have intercepted large quantities of groundwater. This work is still in progress. It is estimated that six to ten additional horizontal drains will be installed under this contract. Dewatering activities at the Pinole slipout are shown by Figure 14.

III. LABORATORY TESTING AND STABILITY ANALYSIS

A laboratory testing program was begun upon receipt of the first soil samples from the field. The major portion of the program, which included routine classification tests, was devoted to determining soil strength by triaxial compression tests. These tests, scheduled daily, were based on a continuing review of boring logs and samples as they were received. Most of the tests were conducted on unconsolidated-undrained specimens although several consolidated-undrained, consolidated-drained, and unconfined compression tests were included in the program. The unconsolidated-undrained tests were about equally divided between laboratory saturated specimens (with pore pressure measurement) and specimens at field moisture content. Because of time limitations, testing was restricted primarily to samples from six borings nearest the axis of mass movement.

An examination of data from the field exploration and laboratory testing programs shows a complex, nonuniform, subsurface stratification consisting of firm to stiff silty clays, sands with small amounts of gravel, and sandy clays. Predominant materials appear to consist of sandy silty clays with the three fractions being present in more or less equal percentages. The sands appear to be localized and vary the greatest in stratum or deposit thickness.

With the exception of the sands and a few silty clay samples bordering the sands, the soils were only damp to slightly moist and showed no evidence of free or excesss water despite the fact free water was encountered in practically all borings. This is undoubtedly the result of the impermeable nature of the clays which created an artesian condition in the strata above the sandy aquifer layer. The lack of excess water is also reflected in triaxial compression tests on specimens at field moisture contents, compressive strengths of which generally ranged between 2 and 6 TSF (tons per square foot). The same materials, when saturated and tested under undrained conditions, exhibited compressive strengths in the order of 0.7 to 1.5 TSF. It is therefore considered that failure resulted from a strength loss due to excess hydrostatic pressure within the saturated silty clay bordering the sandier deposits that functioned as aquifers. The depths at which free water was encountered varied from 20 to 35 feet between borings but the most frequent depths were 23 to 28 feet below original ground surface. Almost without exception, water rose in the borings. At the toe of the detour embankment this rise was as much as 10 to 15 feet.

A stability analysis was performed by electronic computer on a section through the axis of movement. Assumptions included a circular failure surface and $\emptyset = O$ conditions for soil strength. Since the foundation soils in the failure zone were saturated and in an undrained condition, the $\emptyset = O$ analysis was considered appropriate. Seepage forces were not introduced in the analysis because of the apparent low flow gradient which would result in very small increases in driving forces. As an aid in assigning soil strength parameters, a simplified soil profile was constructed, as shown in Figure 15. Strength values representative of the saturated unconsolidated-undrained conditions were selected from test data and are shown on Figure 15.

[illegible] upon receipt of [illegible] the program [illegible] limited to determining [illegible] these tests, selected daily [illegible] samples as they were [illegible] unconsolidated-undrained [illegible] consolidated-drained [illegible] program. The [illegible] equally divided between labora- [illegible] accurate measurements and expedi- [illegible] time limitations, testing was [illegible] from six borings near the axis of mass [illegible]

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[illegible] of the sands and a few silty clay samples [illegible] were only damp to slightly moist and showed [illegible] despite the fact free water was encountered [illegible] This is undoubtedly the result of the [illegible] an artesian condition in the strata [illegible] as water is also reflected in typical [illegible] field moisture contents, compressive [illegible] between 2 and 6,150 (compressive [illegible] when saturated and tested under [illegible] strength in the order of 1,700 to 5,150 [illegible] failure resulted in loss of strength due to [illegible] the saturated silty clay beds acting as [illegible] as aquifers. The depth at which free water [illegible] from 20 to 25 feet between borings [illegible] feet below original ground surface [illegible] in the borings. At the top of the [illegible] 75 feet.

[illegible] was performed by electronic computer [illegible] including a circular [illegible] strength. Since the [illegible] saturated and in an undrained condition, these [illegible]

Three trial circles were used as starting points in a computer search program in which the radius is first varied to determine a minimum safety factor and then the circle center is varied to find another minimum safety factor. For the conditions analyzed, a minimum safety factor of 1.006 was obtained for the circle shown on Figure 15. A value of 1.009 was obtained for the same center but with a five foot longer radius.

The mode of failure appears to have been more of the translational type than circular, especially through the lower area of movement. The use of a circular failure surface as an approximation is believed to have been justified since the critical circle was fairly flat through the zone of original ground and conformed reasonably well to the available inclinometer data. Also the safety factor of unity attested to the reasonableness of strength values used in the analysis.

Stability computations were also performed to determine the effects of possible corrective treatment. One such treatment consists of constructing a berm parallel to and downhill of the existing embankment (as shown in Figure 15) to provide a counterweight against potential future movement. The possibility of constructing a stabilization trench within the slide zone was also considered. Since this construction would require removing a portion of the slide material to a depth of 30 to 40 feet below original ground, a condition of questionable stability would be temporarily created. This condition was analyzed and found to be marginal as shown by the table below, which also includes results of the berm analysis. Strengths used in these analyses were the same as those used in analyzing the failure and do not reflect any improvement that would be expected upon providing adequate subsurface drainage.

Summary of Stability Analysis

Condition Analyzed	S.F.
Original Embankment Failure	1.006
Reconstructed Embankment with Berm:	
20-foot high berm	1.10
30-foot high berm	1.34
40-foot high berm	1.66
Embankment at full stabilization trench excavations	1.04

IV. CONCLUSIONS

Analysis of data resulting from visual observations, borings, field measurements and laboratory tests result in the following conclusions with respect to the fill slipout on Interstate 80 near Pinole.

1. The embankment slipout was a result of a loss of strength of foundation soils due to an unprecedented rise in water table elevation. This rise was due in part to the severest series of winter rainstorms since the highway embankment was constructed both in terms of total precipitation in number of rain days during the winter season. Another factor which may have contributed to the rise in water table elevation, was the construction of subdivision fills near the west right of way line in the slide area 8 years after construction of the highway embankment. This non-highway associated construction may have constricted drainage or pinched off natural drainage aquifers resulting inevitably in increase in water table elevation (The fact that the highway embankment suffered no distress in the winter of 1957-58 which was very nearly as severe as that of 1968-69, provides further indication of the possible detrimental effect of the subdivision fill construction).

2. While the original exploration conducted in 1954 was quite extensive for the project as a whole, only a limited effort was expended in the immediate area of the slipout which at that time did not appear to present any special problems. In the two borings which were taken, groundwater was not encountered. Competent foundation materials were noted at relatively shallow depths. These findings were later substantiated by the firm of Woodward-Clyde and Associates during their exploration for Tara Hills Subdivision Unit #8 immediately to the west of the fill slipout in 1964.

3. The soil profile is a complex nonuniform surface stratification consisting of firm to stiff sandy clays, sands with small amounts of gravel and sandy clays. Sandy, silty clays with three fractions in approximately equal percentages appear to predominate. The movement, which was more translational than rotational in nature, probably occurred near the surface between the tan and gray sands, silt and clay and yellowish and gray silty clay 20 to 25 feet below original ground surface.

4. The embankment was a dry, well compacted, high quality material as indicated by the near vertical nature of the larger slide scarp.

... the following conclusions, with ...

... result of a loss of strength of ... in water table elevation. This ... winter rainstorm - since the ... in terms of total precipitation in ... season. Another factor which may ... elevation was the construction of ... way line in the slide area 8 years ahead ... This highway was obtained ... or patched off natural drainage ... increase in water table elevation (The fact that ... distress in the winter of 1957-58 which ... 1968-69, provides further indication of ... of the subdivision fill construction).

... the original exploration conducted in 1954 was quite ... as a whole, only a limited effort was expended at ... which at that time did not appear to present ... borings which were taken, groundwater was ... foundation materials were noted at relatively ... were later substantiated by the firm of ... exploration for Tara Hills ... of the lift slope ...

... surface stratifica- ... with small amounts of ... with three fractions in approxi- ... The movement, which was more ... probably occurred near the surface between ... yellow and gravelly clay ...

... dry, well compacted, high quality ... of the largest slide scarp.

V. CORRECTION

For a permanent correction, two lines of vertical wells will be drilled approximately parallel to the roadway as shown by Figure 5. One line will be located just outside the toe of the present detour fill while the other will be in the vicinity of the original downhill embankment toe. The wells will be three feet in diameter, spaced at four foot centers and belled to five feet in diameter to form a continuous inner connection. The wells will be backfilled with clean gravel. In order to accomplish their purpose of intercepting and collecting groundwater in the embankment foundation area, it is estimated that well depths of 45 to 50 feet will be required. It is necessary that water collected by the wells be drained away if a permanent lower water table is to be maintained. The uphill wells will be intercepted by horizontal drains to be installed from suitable headings. The lower wells will be drained by a pipe extending transverse to the line of wells and terminating at the drainage facility in the low area near the housing development. The pipe will be laid in a trench to be excavated and bedded with gravel beginning at an elevation of +123 at the drainage outlet. The pipe will be perforated and laid on as flat a grade as practical in order to tap the lower wells at as great a depth as possible. It appears that each line of wells will be 500 to 700 feet in length with midpoints near Station 207.

To provide increased stability of the embankment, a berm ±25 feet high will be constructed downhill of the roadway and extended laterally a distance of ±50 feet beyond the downhill limit of movement. Longitudinally, the berm will key into the existing subdivision embankments which converge northerly of the original embankment toe. Standard embankment compaction will be required. Stability analysis indicates a factor of safety in the order of 1.4 for the roadway embankment with the use of such a berm. The possibility of added embankment width due to widening of the existing roadway will create no stability problems.

The above correction plan has been submitted to the Bureau of Public Roads.

TABLE I

Precipitation, in inches, at Crockett (1) and Richmond (2)

(1) U.S.W.B. Station No. 2177, 38° 03' N, 122° 13' W, Elevation 9

(2) U.S.W.B. Station No. 7414, 37° 56' N, 122° 21' W, Elevation 55

Tabulation includes:

Monthly precipitation amounts, July 1958 through April 1969.
Seasonal precipitation totals, for season ending June 30.
Monthly and seasonal average precipitation, based on 1951-1968 reco[illegible]
Monthly and seasonal normal precipitation, based on 1931-1960 reco[illegible]
for Crockett only. Richmond Station opened December 1950.

(1) Crockett

	JAN.	FEB.	MAR	APR	MAY	JUN	JUL	AUG	SEPT	OCT	NOV	DEC	SEAS[illegible]
1958							.05	.12	.04	.18	.01	1.18	
1959	2.87	5.71	.28	.60	.01	-	-	-	1.45	-	-	1.67	11.0[illegible]
1960	3.48	3.76	1.75	.66	.48	-	-	-	-	.32	4.16	1.61	13.2[illegible]
1961	2.11	1.38	2.16	1.25	.15	.03	-	.06	.30	.02	3.51	1.60	13.1[illegible]
1962	.64	6.56	3.38	.27	-	-	-	.05	.01	8.88	.84	2.18	16.[illegible]
1963	3.78	3.72	4.58	4.20	.59	.05	-	.02	.10	1.99	3.98	.47	28.[illegible]
1964	3.77	.12	1.24	.28	.29	1.31	-	.02	-	1.55	2.35	5.86	13.5[illegible]
1965	5.24	.45	1.79	3.18	-	-	-	.39	-	.05	5.01	4.77	20.44
1966	2.69	2.76	.33	.54	.16	.15	.07	.12	.36	-	4.88	3.53	16.85
1967	9.94	.37	5.05	4.88	.15	.76	-	-	.04	.93	1.61	2.00	30.11
1968	6.14	2.22	2.94	.47	.20	-	-	.02	-	.54	3.24	4.61	16.55
1969	7.63	5.67	1.38	1.61									>24.7[illegible]
'51-68 Average	4.20	2.73	2.22	1.69	.42	.19	.01	.05	.19	1.10	2.27	3.47	18.54
'31-60 Normal	3.55	3.26	2.36	1.30	.55	.14	.01	.02	.11	.82	1.67	3.62	17.41

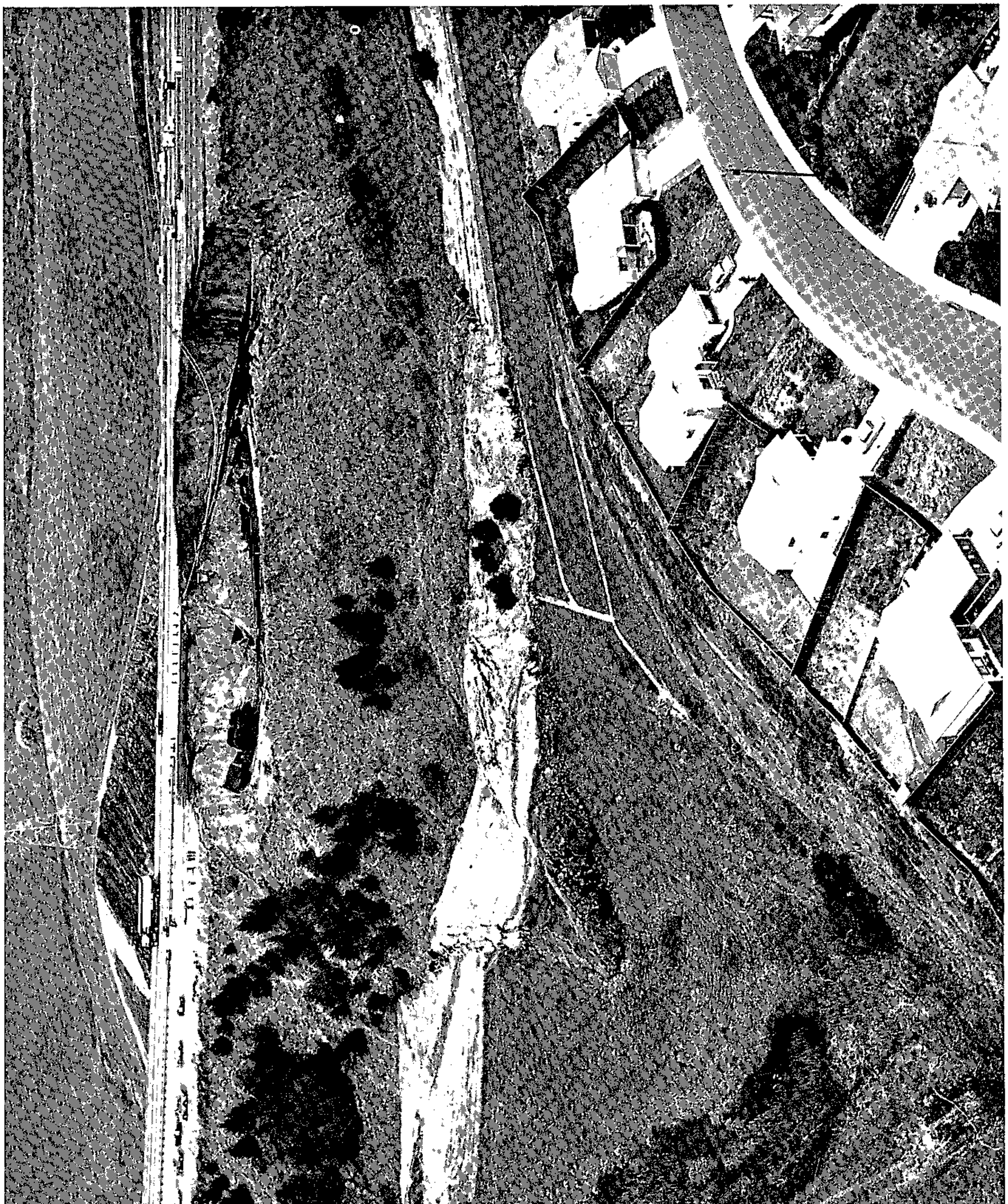
(2) Richmond

1958							.03	-	.04	.16	.03	1.74	
1959	4.15	5.39	.77	.51	-	-	-	-	2.74	-	-	1.93	12.82
1960	4.32	4.14	2.55	1.29	.37	-	-	-	-	.26	3.43	2.61	17.34
1961	1.91	1.51	2.79	1.14	.35	-	-	.05	.29	.05	4.20	2.32	14.00
1962	1.39	7.18	3.76	0.24	-	-	-	.10	.08	7.38	.99	3.48	19.48
1963	4.20	3.47	4.20	5.14	.50	-	-	.03	.13	1.82	3.81	.49	29.54
1964	3.90	.23	1.40	.37	.22	.79	-	.01	-	1.60	4.10	6.93	13.19
1965	4.53	1.24	1.61	3.47	-	-	-	.36	-	.10	4.99	3.70	23.49
1966	4.76	3.01	.51	.37	.17	.06	.05	.08	.14	-	5.93	4.64	18.03
1967	8.21	.38	5.32	5.14	.07	1.15	-	-	.02	.38	1.02	2.40	31.11
1968	5.20	3.21	3.60	.46	.18	-	-	.13	-	1.07	3.05	5.69	16.47
1969	8.19	6.53	1.50	2.05									>28.21
'51-68 Average	4.82	3.13	2.65	1.99	.45	.18	.01	.06	.32	1.09	2.54	4.42	21.66

Source: U.S.Dept. of Commerce "Climatogical Data" (Normals and monthly precipitation through February 1969).

U.S. Weather Bureau, San Francisco (March and April, 1969 precipitation).

Figure 1



PINOLE SLIPOUT

Figure 2

COMPARATIVE PRELIMINARY FOUNDATION INVESTIGATION

ROAD 04-CC-80

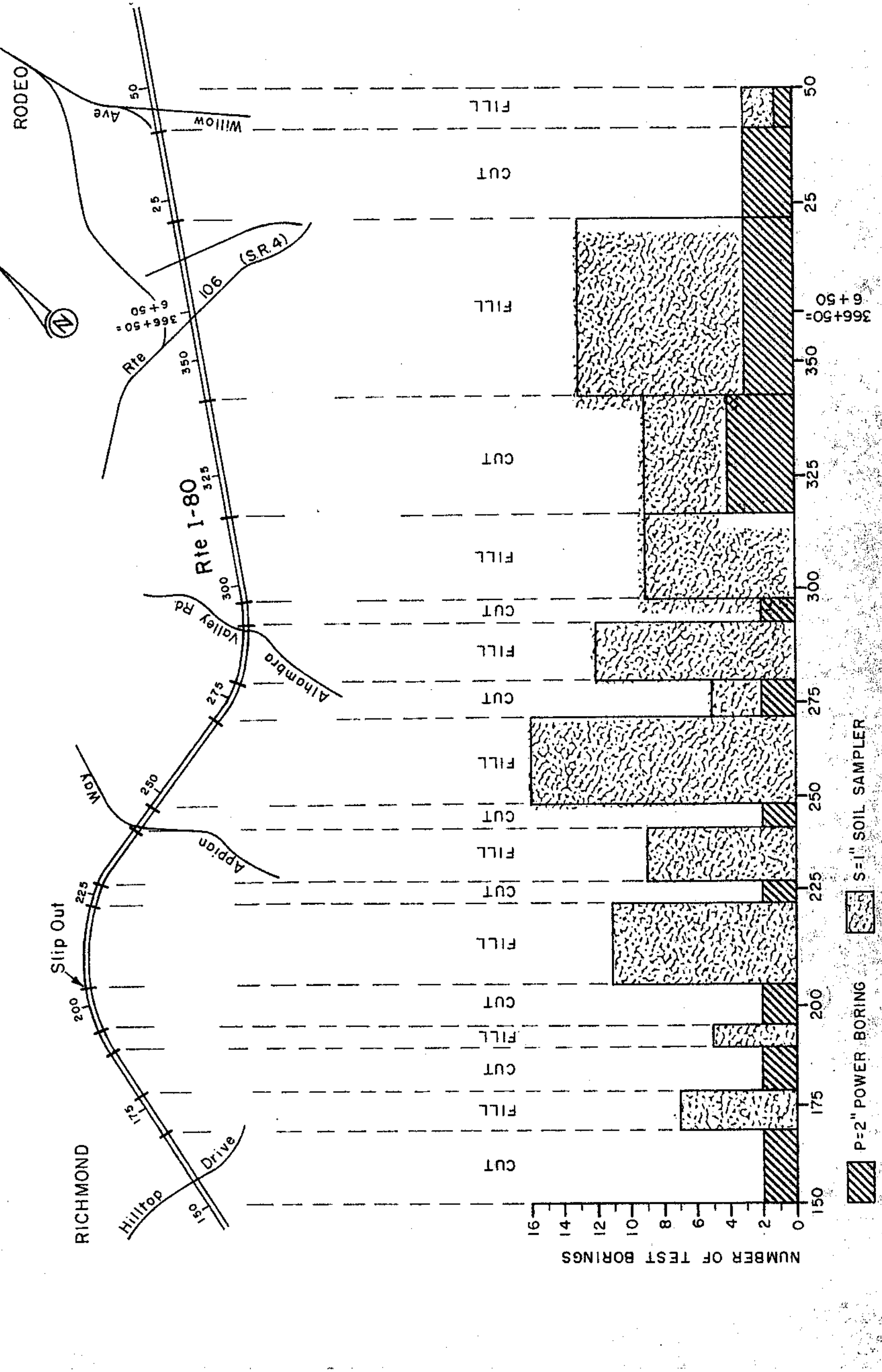
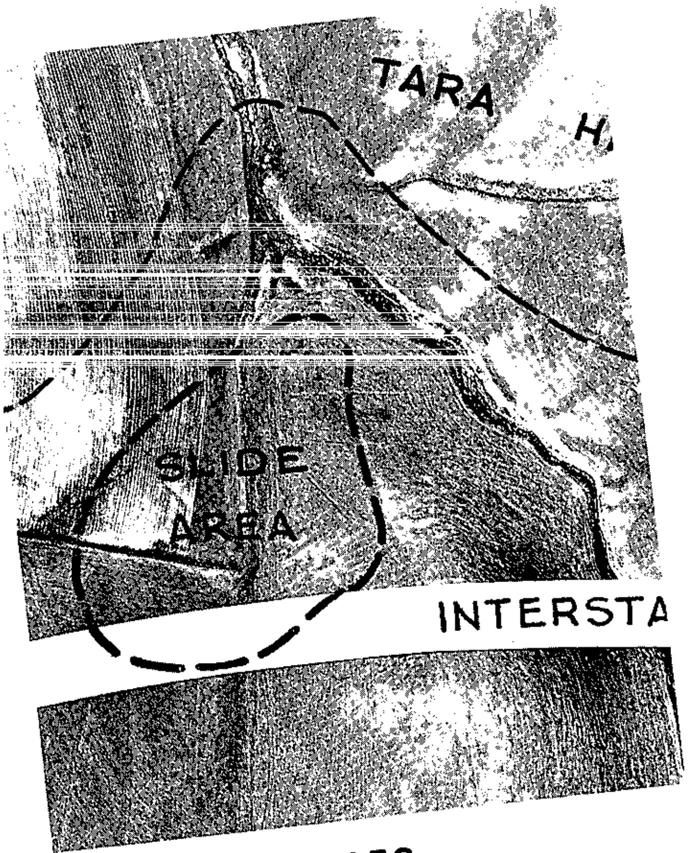
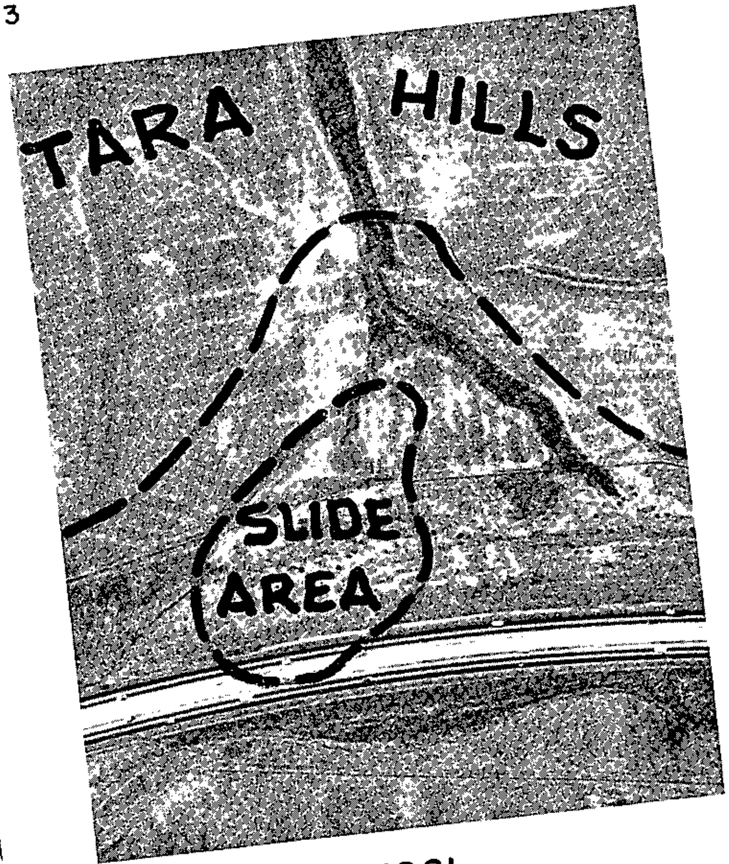


Figure 3



1956



1961

1965

1969

AERIAL VIEWS OF SLIDE AREA (1956-1969)

Figure 4

TOTAL NUMBER OF DAYS WITH
RAINFALL EXCEEDING 0.10 INCH

OCTOBER TO MARCH 1953–1969
Richmond, Calif.

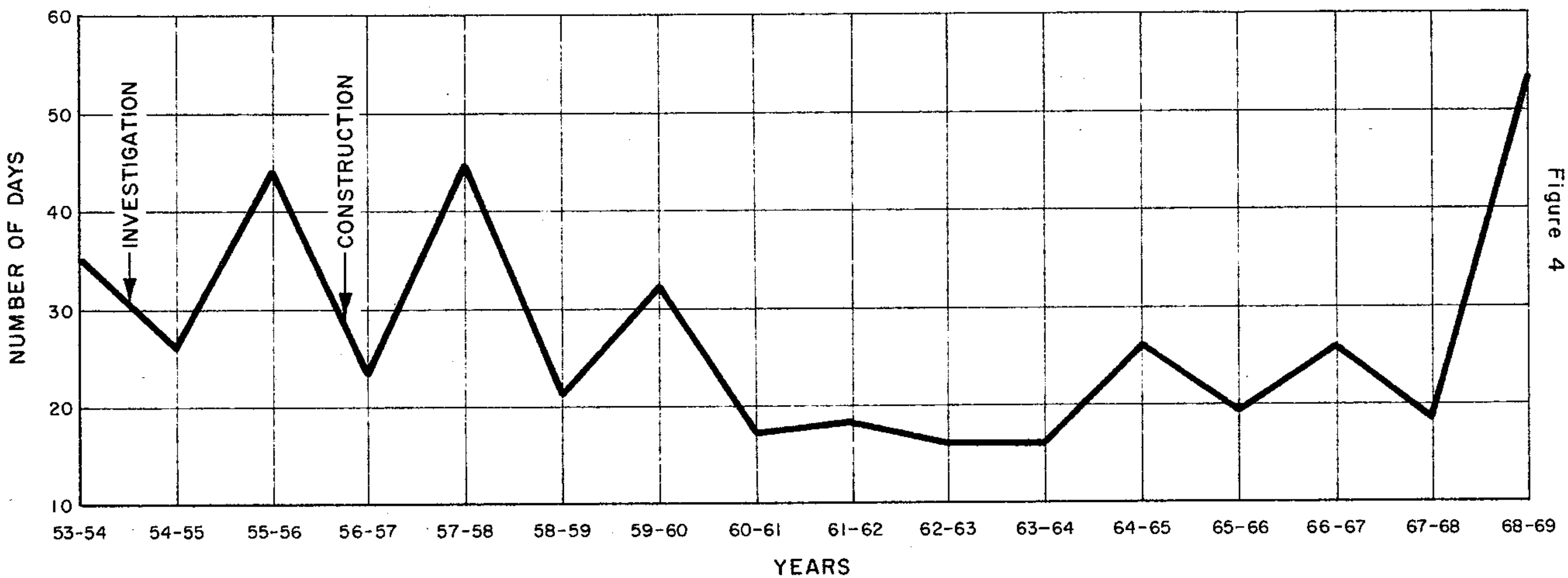
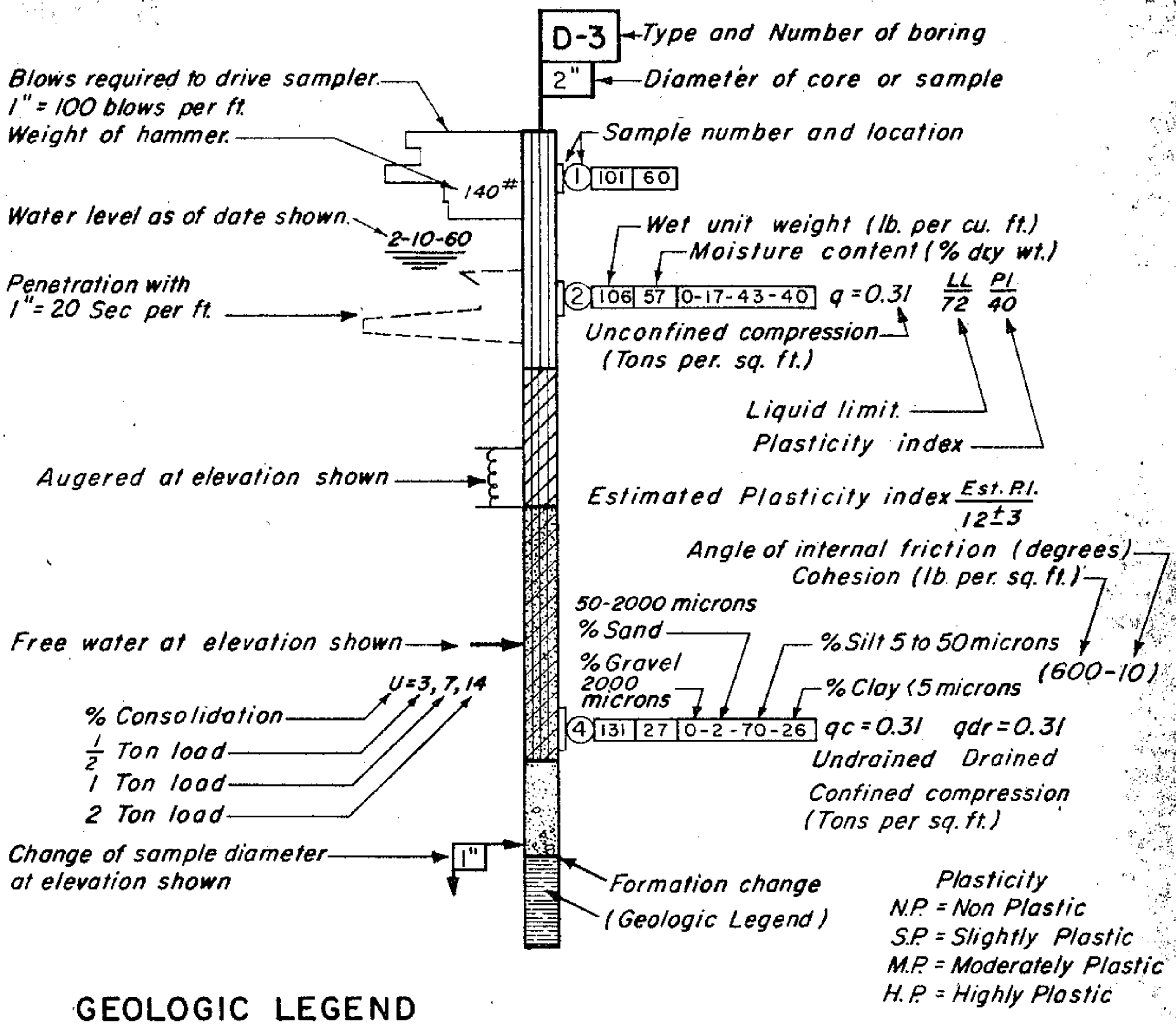


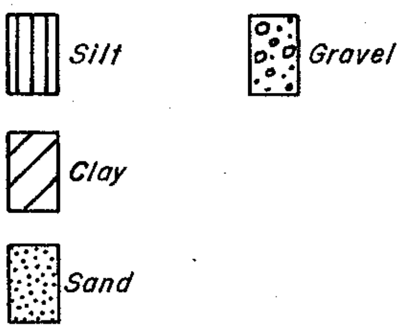
Figure 6

BORING LEGEND

CROSS-SECTION & PROFILE SHEETS



GEOLOGIC LEGEND



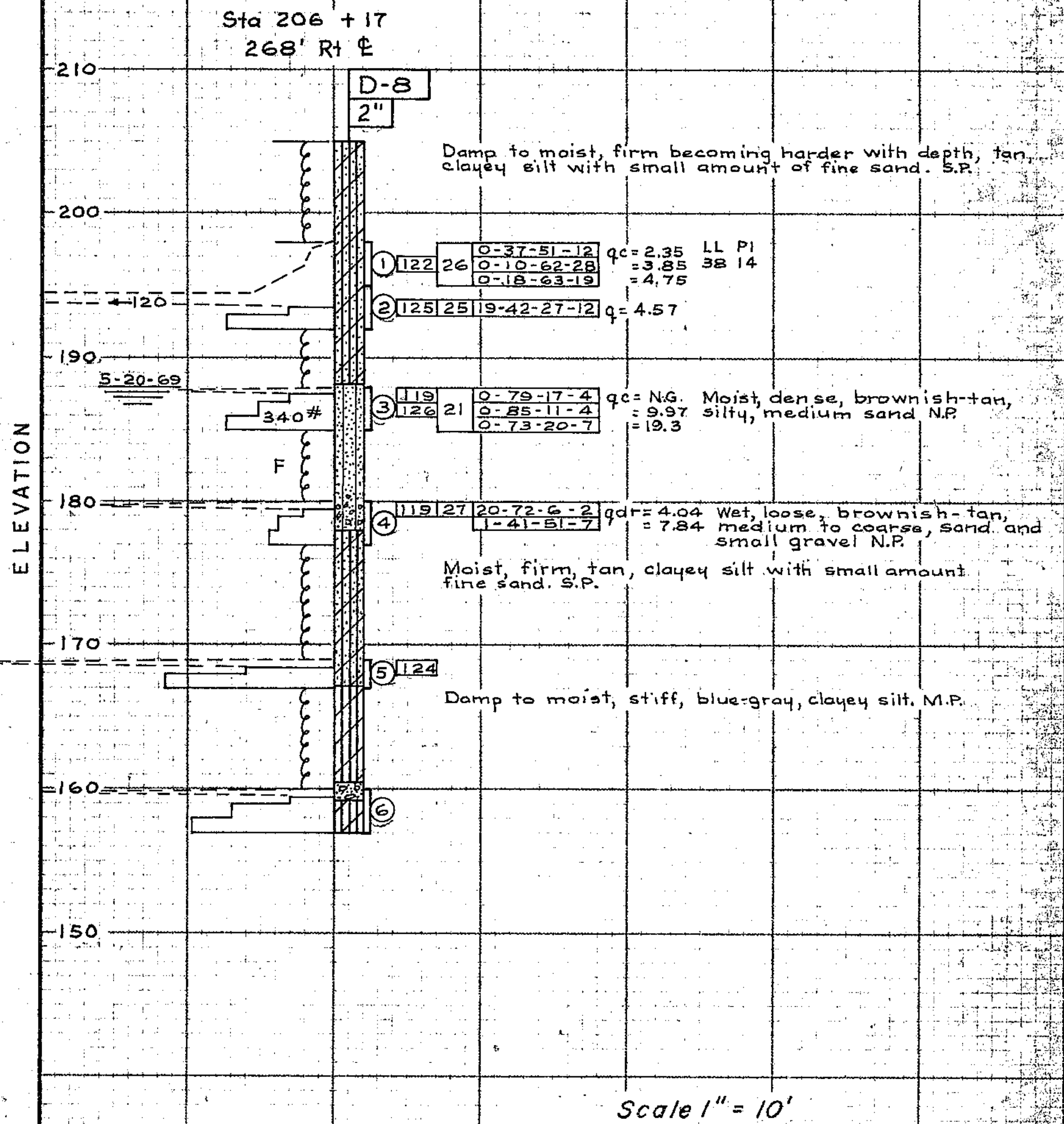
STATE OF CALIFORNIA — HIGHWAY TRANSPORTATION AGENCY
DEPARTMENT OF PUBLIC WORKS
DIVISION OF HIGHWAYS
MATERIALS AND RESEARCH DEPARTMENT

FOUNDATION INVESTIGATION

ROAD 04-CC-80

PINOLE SLIPOUT

DATE	SUBMITTED BY	DWG. NO.
6-13-69	[signature] SR. MAT'LS. & RES. ENG'R.	2082 A
DR. BY MI, RS, DD / CK. BY RP	APPROVED BY: [signature] ASST. MAT'LS. & RES. ENG'R. — FDN	SHEET NO. OF SHTS.



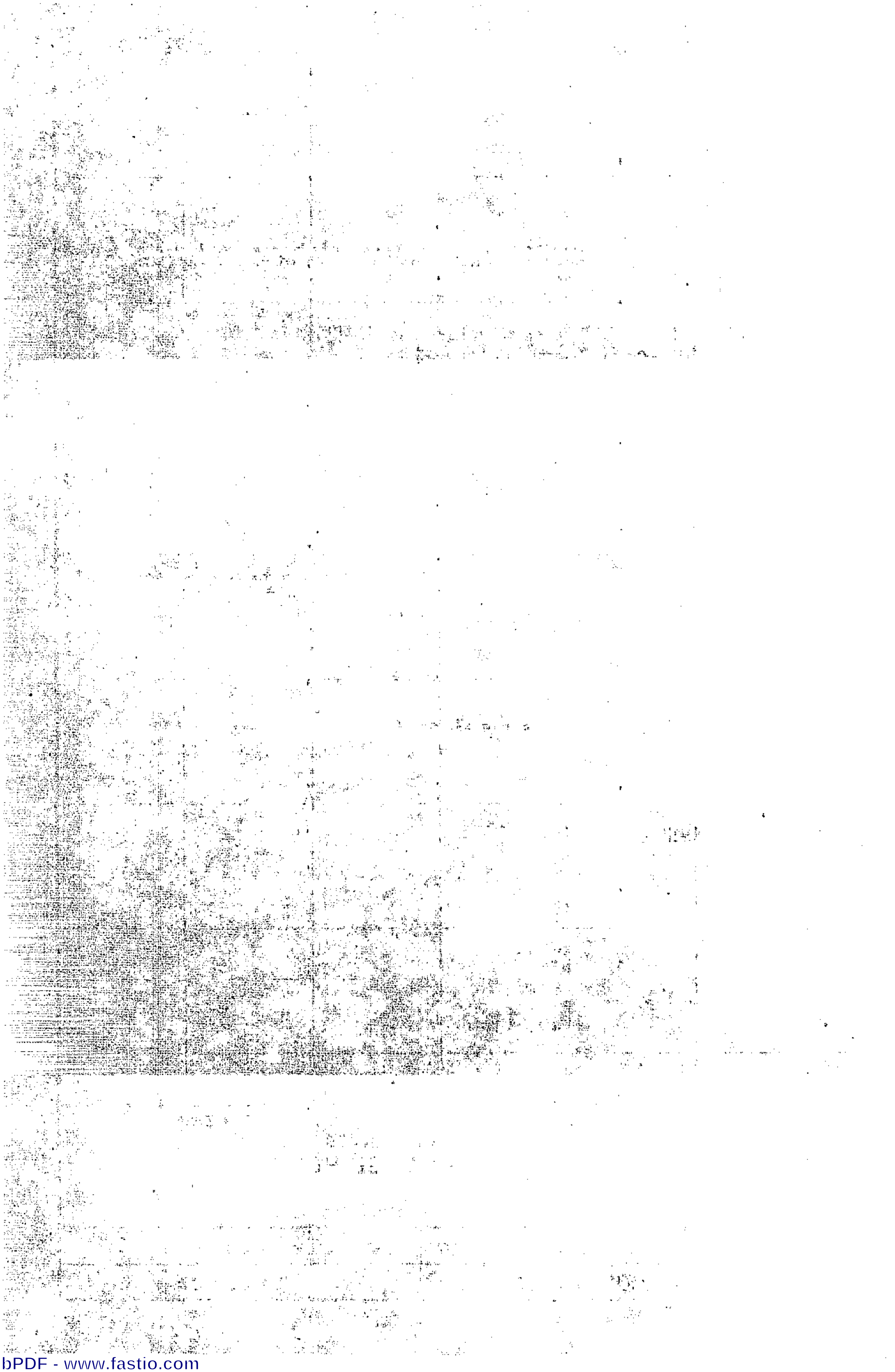
Scale 1" = 10'

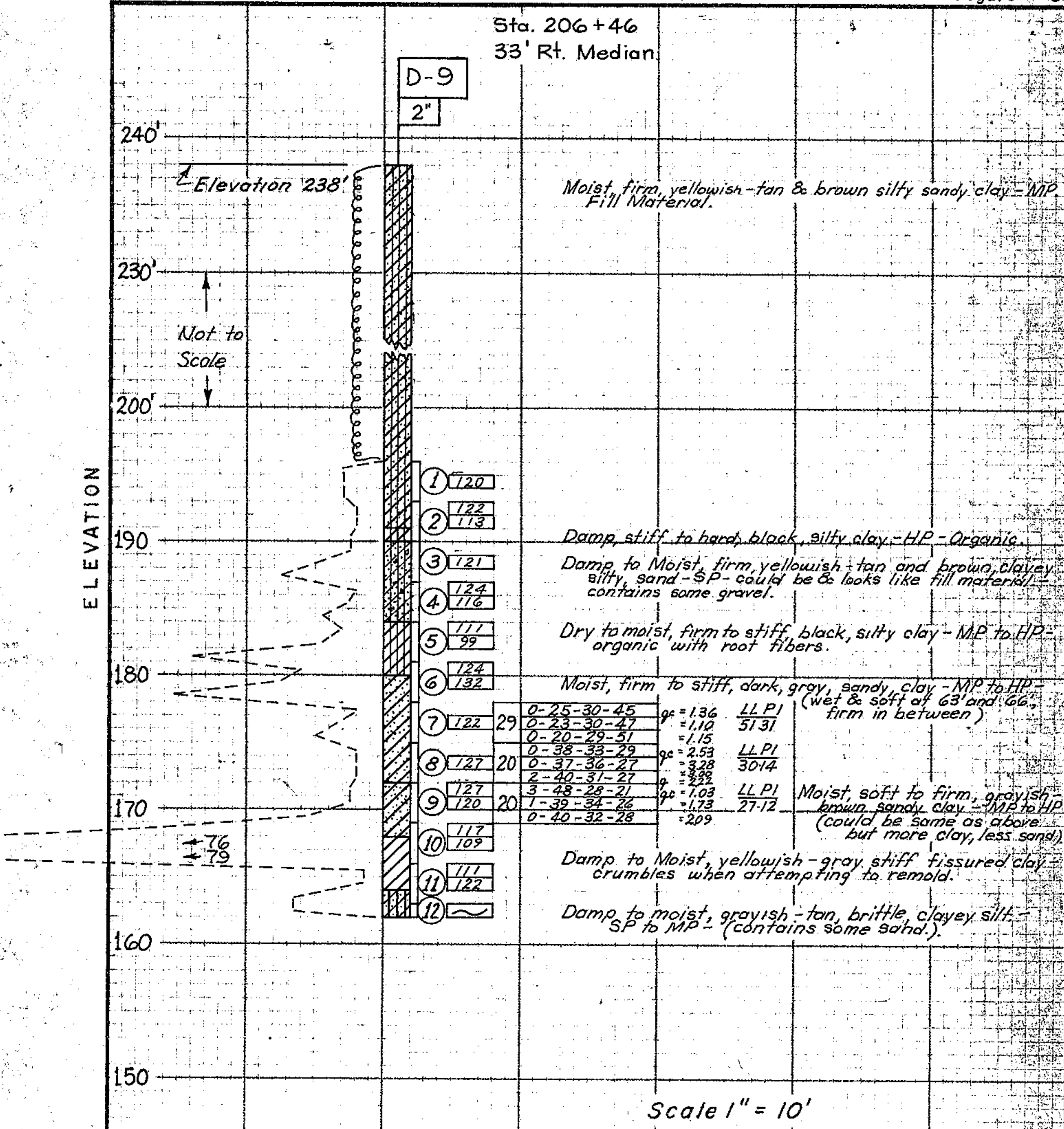
MATERIALS AND RESEARCH DEPARTMENT
FOUNDATION INVESTIGATION

PINOLE SLIPOUT

Profile of Borings D-8

DATE	DIST	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/13/69	04	CC	80		2082A	of





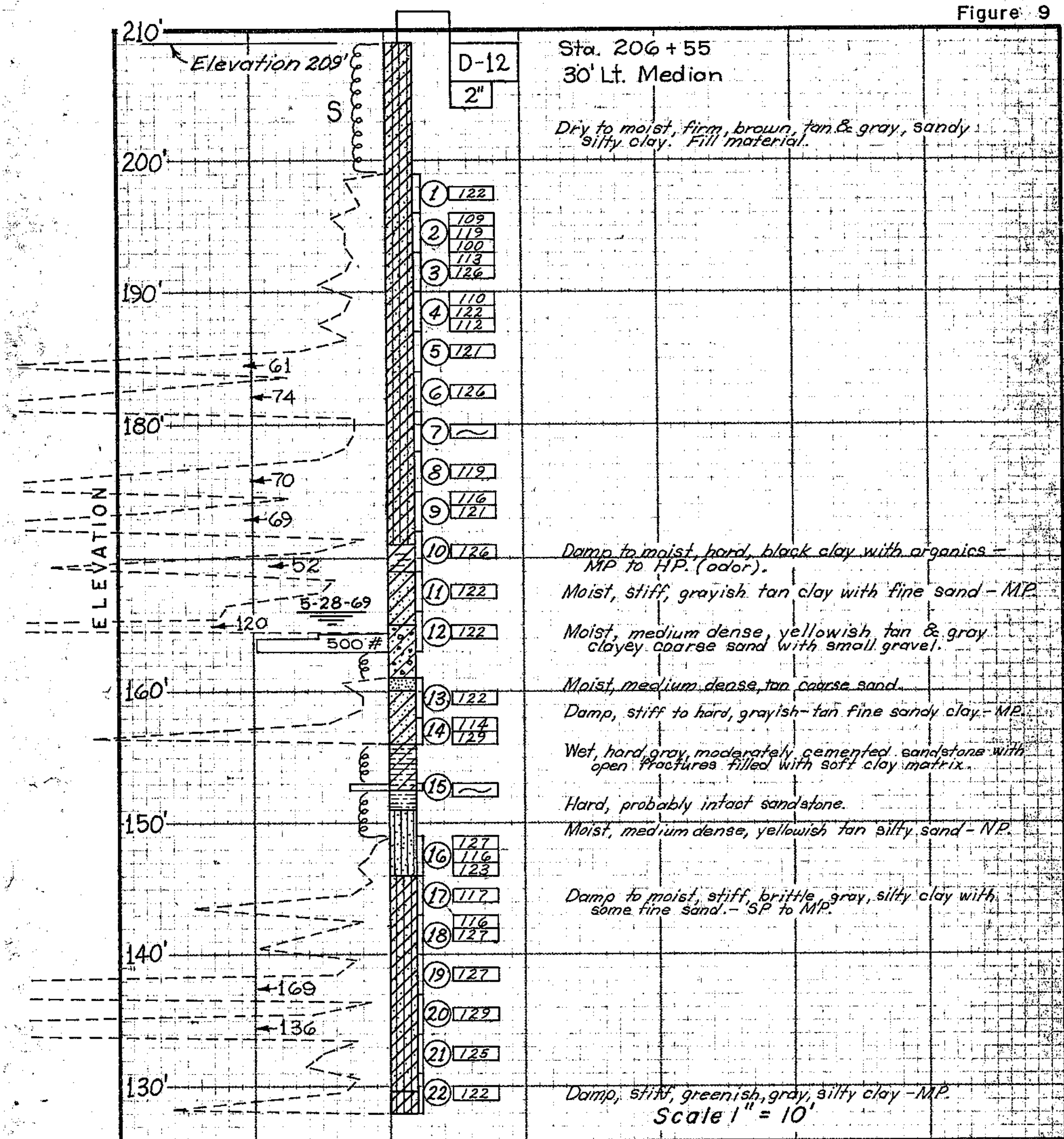
MATERIALS AND RESEARCH DEPARTMENT
FOUNDATION INVESTIGATION

PINOLE SLIPOUT

Profile of Boring D-9

DATE	DIST	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/13/69	04	CC	80		2082A	of

Figure 9



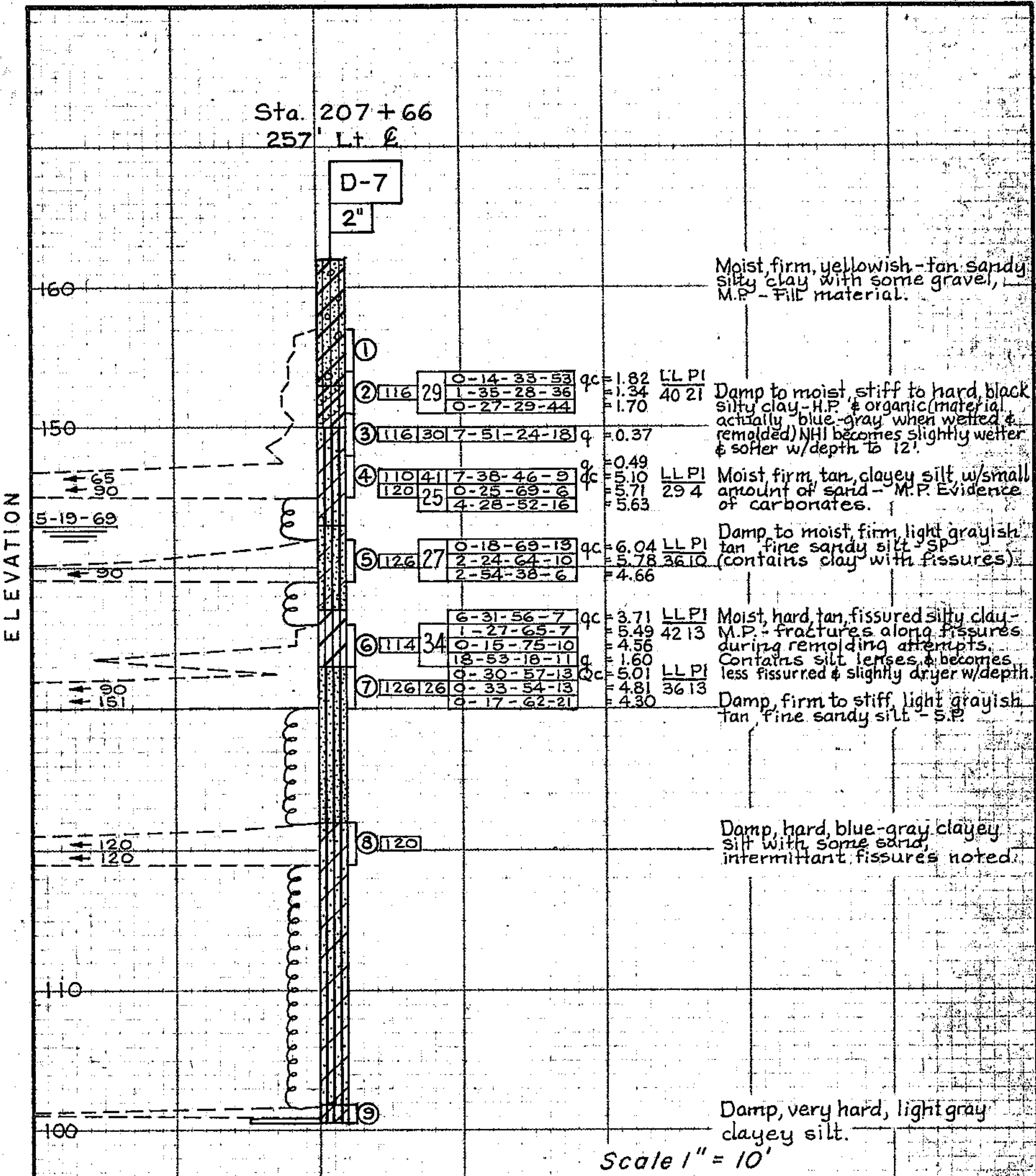
MATERIALS AND RESEARCH DEPARTMENT

FOUNDATION INVESTIGATION

PINOLE SLIPOUT

Profile of Boring D-12

DATE	DIST	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/13/69	04	CC	80		2082A	of



MATERIALS AND RESEARCH DEPARTMENT

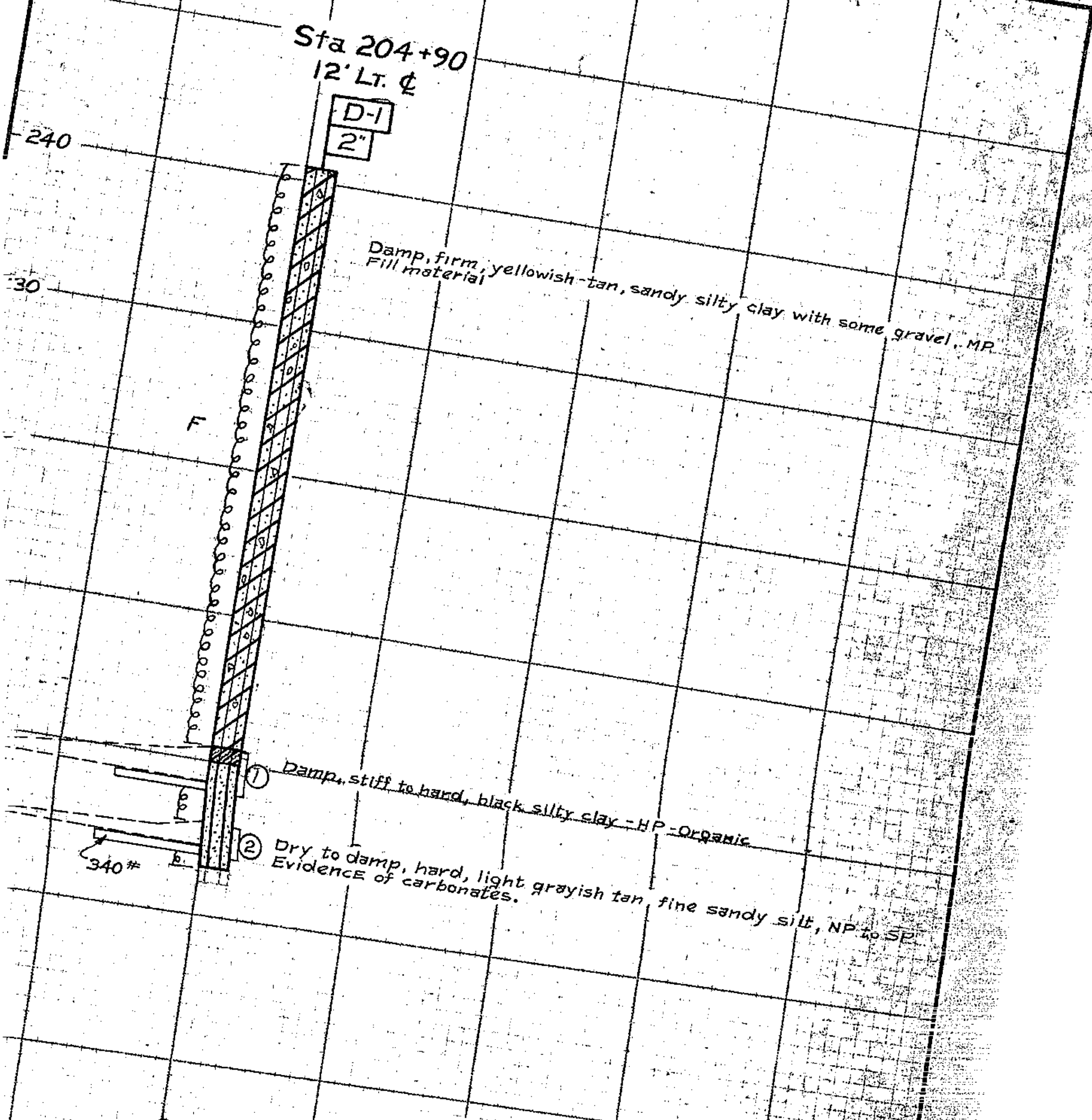
FOUNDATION INVESTIGATION

PINOLE SLIPOUT

Profile of Boring D-7

DATE	DIST	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/13/69	04	CC	80		2082A	of

Figure 11



Scale 1" = 10'

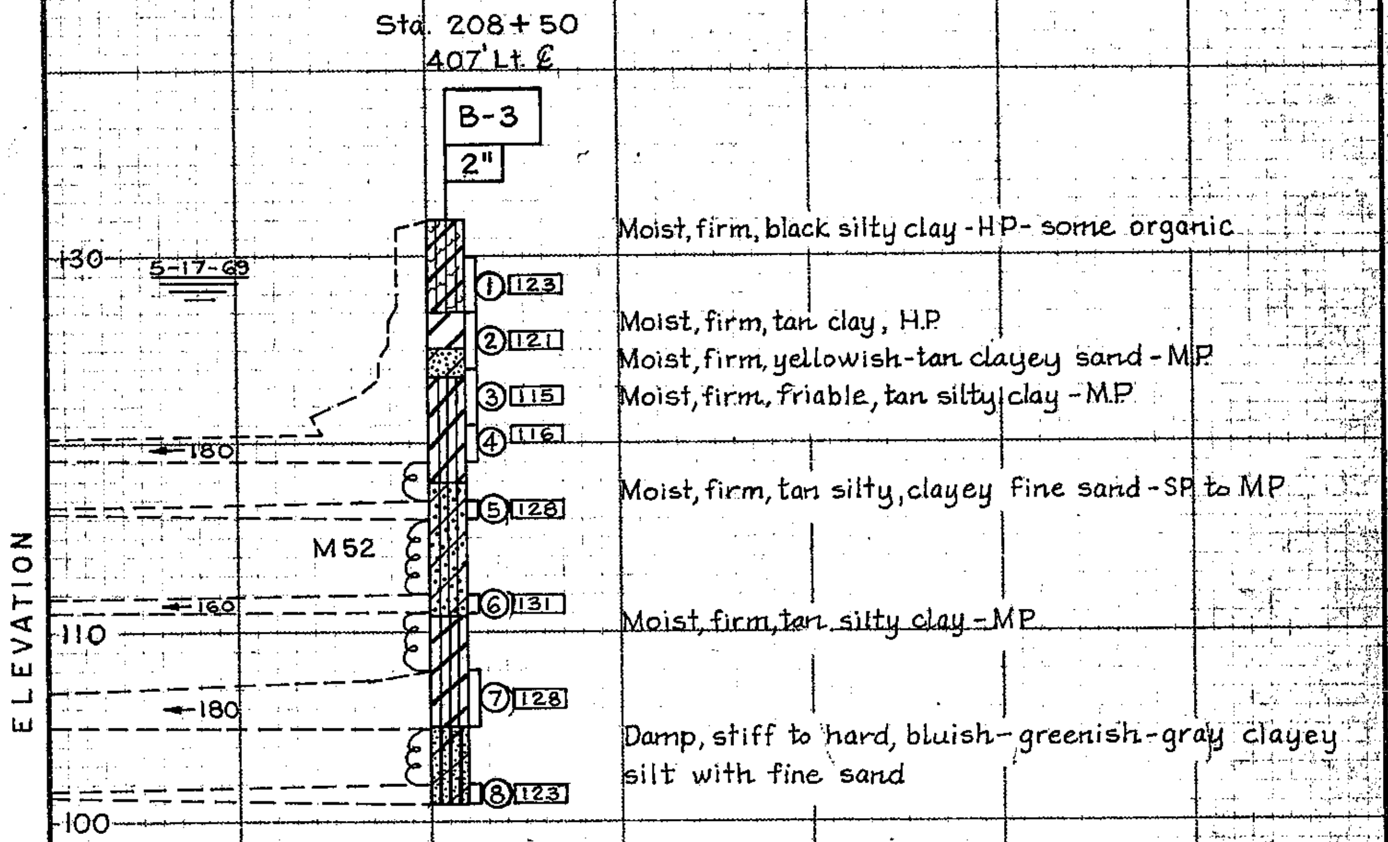
MATERIALS AND RESEARCH DEPARTMENT
FOUNDATION INVESTIGATION

PINOLE SLIPOUT

Profile of Boring D-1

DATE	DIST	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/13/69	04	CC	80		2082A	of

Figure 12



Scale 1" = 10'

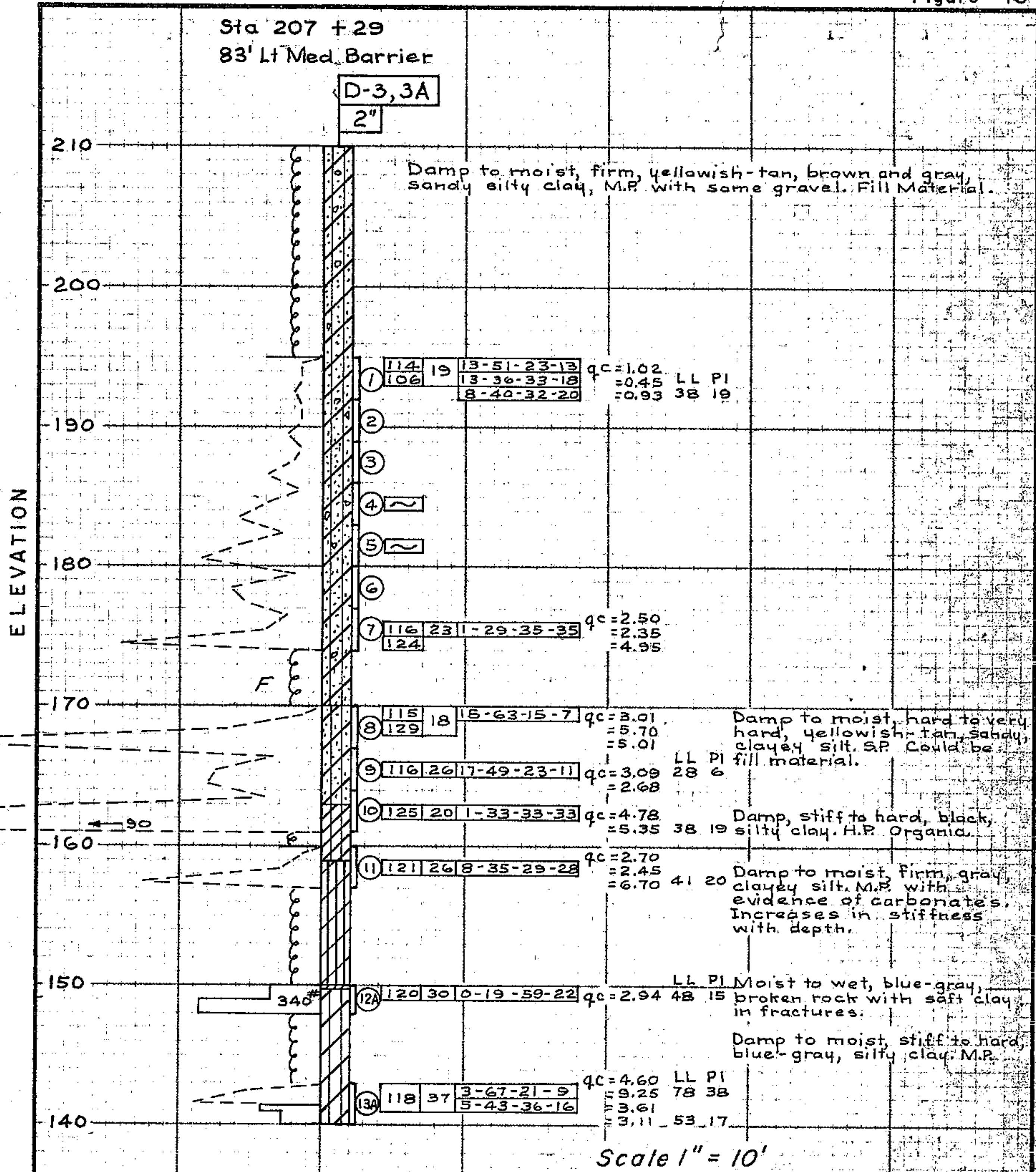
MATERIALS AND RESEARCH DEPARTMENT
FOUNDATION INVESTIGATION

PINOLE SLIPOUT

Profile of Boring B-3

DATE	DIST	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/13/69	04	CC	80		2082A	of

Figure 13

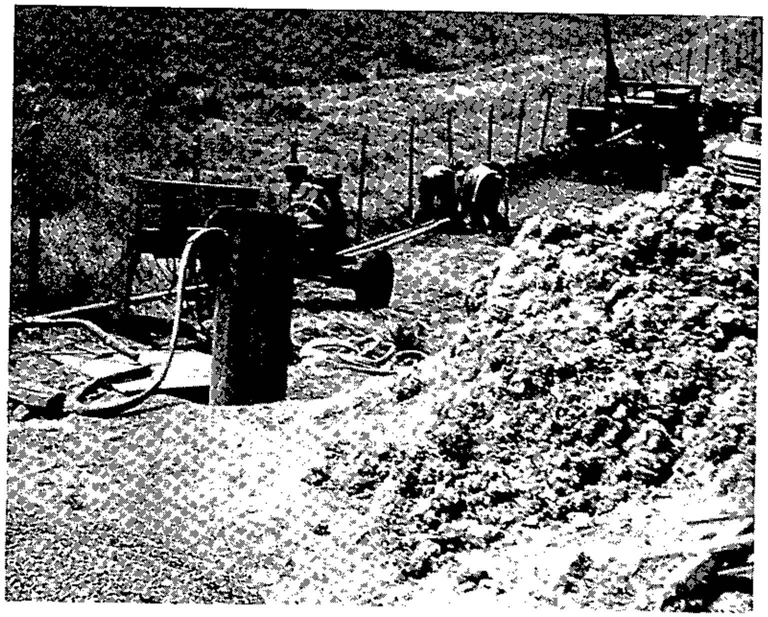


MATERIALS AND RESEARCH DEPARTMENT
FOUNDATION INVESTIGATION
PINOLE SLIPOUT

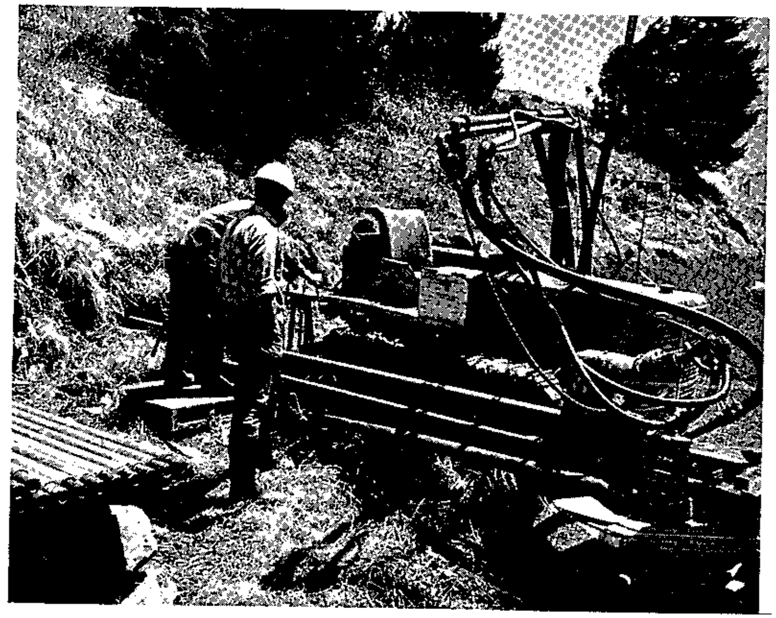
Profile of Borings D-3, 3A

DATE	DIST	COUNTY	ROUTE	POST MILE	DWG. NO.	TOTAL SHEETS
6/13/69	04	CC	80		2082A	of

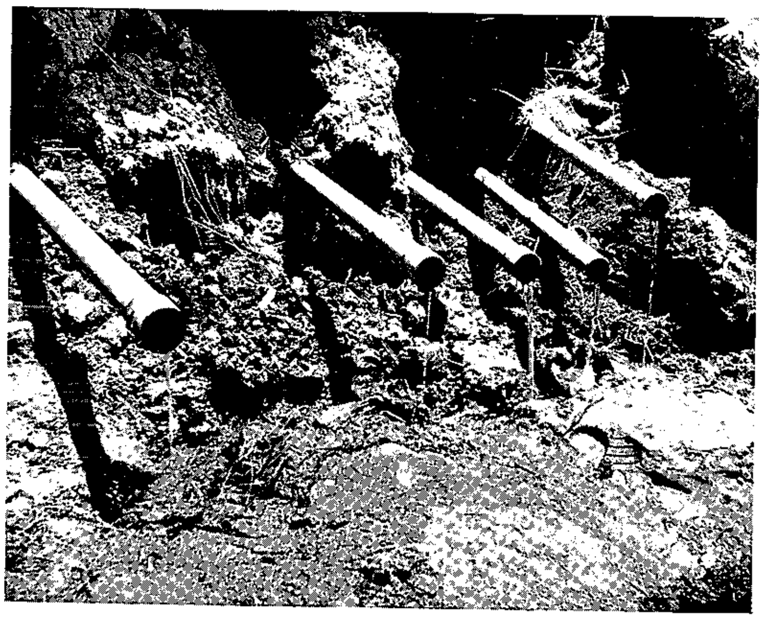
Figure 14



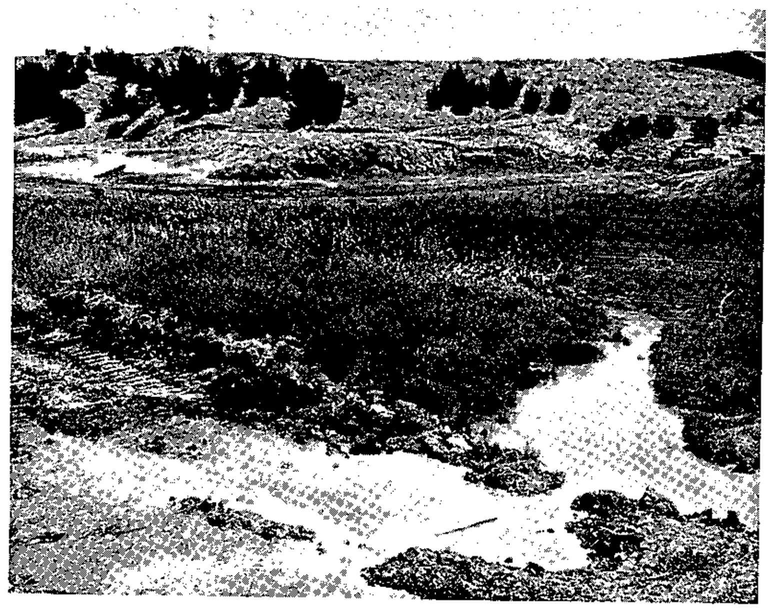
Pumping from cased 3' Dia. Auger Holes



Horizontal Drain Installation



Horizontal Drains



Free Water at Slide Toe

DEWATERING ACTIVITIES AT PINOLE SLIPOUT